Chapter 15
Dam Freeboard Requirements

15-1. Basic Considerations

a. Freeboard. Freeboard protects dams and embankments from overflow caused by wind-induced tides and waves. It is defined as the vertical distance between the crest of a dam and some specified pool level, usually the normal operating level or the maximum flood level. Depending on the importance of the structure, the amount of freeboard will vary in order to maintain structural integrity and the estimated cost of repairing damages resulting from overtopping. Riprap or other types of slope protection are provided within the freeboard to control erosion that may occur even without overtopping.

b. Estimating freeboard. Freeboard is generally based on maximum probable wind conditions when the reference elevation is the normal operating level. When estimating the freeboard to be used with the probable maximum reservoir level, a lesser wind condition is used because it is improbable that maximum wind conditions will occur simultaneously with the maximum flood level. A first step in wave height determinations is a study of available wind records to determine velocities and related durations and directions. Three basic considerations are generally used in establishing freeboard allowance. These are wave characteristics, wind setup, and wave runup.

c. Further information. The Corps of Engineers Coastal Engineering Research Center (CERC) has developed criteria and procedures for evaluating each of the above areas. The primary references are EM 1110-2-1412 and EM 1110-2-1414. The procedures presented in these manuals have received general acceptance for use in estimating freeboard requirements for reservoirs.

d. Applications. In applications for inland reservoirs, it is necessary to give special consideration to the influences that reservoir surface configuration, surrounding topography, and ground roughness may have on wind velocities and directions over the water surface. The effects of shoreline irregularities on wave refraction and influences of water depth variations on wave heights and lengths must be accounted for. Although allowances can only be approximated, the estimates of wave and wind tide characteristics in inland reservoirs can be prepared sufficiently accurate for engineering purposes.

15-2. Wind Characteristics over Reservoirs

a. General. The more violent windstorms experienced in the United States are associated with tropical storms (hurricanes) and tornadoes. Hurricane wind characteristics may affect reservoir projects located near Atlantic and Gulf coastlines, but winds associated with tornadoes are not applicable to the determination of freeboard allowances for wave action. In mountainous regions, the flow of air is influenced by topography as well as meteorological factors. These “orographic” wind effects, when augmented by critical meteorological patterns, may produce high wind velocities for relatively long periods of time. Therefore, they should be given special consideration in estimating wave action in reservoirs located in mountainous regions. In areas not affected by major topographic influences, air movement is generally the result of horizontal differences in pressure which in turn are due primarily to large-scale temperature differences in air masses. Wind velocities and durations associated with these meteorological conditions, with or without major influences of local topography, are of major importance in estimating wave characteristics in reservoirs.

b. Isovel patterns. Estimates of wind velocities and directions near a water surface at successive intervals of time, as a windstorm passes the area, may be established by deriving “isovel” patterns. Sequence relations can represent wind velocities at, say, one-half hour intervals during periods of maximum winds, and one-hour or longer intervals thereafter. The “isovel” lines connect points of equal wind speeds, resembling elevation contour maps. Wind directions are indicated by arrows. EM 1110-2-1412, Sections 1.9 and 1.10 describe storms and the storm surge generation process. Figure 1-1a shows an example wind isovel pattern and pertinent parameters.

c. Relation of wind duration to wave heights. If wind velocity over a particular fetch remains constant, wave heights will progressively increase until a limiting maximum value is attained, corresponding approximately to relations dependent on fetch distance, wind velocity, and duration. Accordingly, wind velocity-duration relations applicable to effective reservoir fetch areas are needed for use in computing wave characteristics in reservoirs.

d. Wind velocity-duration relations. In some cases it is desired to estimate wave characteristics in existing reservoirs in order to analyze causes of riprap damage or for other reasons. Wind records, supplemented by meteorological studies are usually required. Data on actual windstorms of record have been maintained at many U.S. Weather Bureau stations. Index values, such as the fastest mile, 1-min average or 5-min average velocities, with direction indications, are usually presented in climatological data publications. Some data collected by other agencies and private observers may be available in published or unpublished form. However, information regarding wind velocities sustained for several hours or days is not ordinarily published in detail. Accordingly, special studies are usually required to determine wind velocity-duration relations applicable to specific effective fetch areas involved in wave computations. Basic records for such studies are usually available from the U.S. Weather Bureau offices or other observer stations. Some summaries of wind velocities over relatively long periods of time have been published by various investigators, and others may be available in project reports related to water resources development.

e. Generalized wind velocity-duration relations. Studies show that maximum wind velocities in one general direction during major windstorms, in most regions of the United States, have averaged approximately 40 to 50 mph for a period of 1 hr. Corresponding velocities in the same general direction for periods of 2 hr and 6 hr have averaged 95 percent and 88 percent, respectively, of the maximum 1-hr average velocity. In EM 1110-2-1414, Figure 5-26 provides the ratio of wind speed of any duration to the 1-hr wind speed. Extreme wind velocities for brief periods, normally referred to as "fastest mile" or 1-min average, have been recorded as high as 150 to 200 percent of maximum 1-hr averages in most regions. In EM 1110-2-1414, Figures 5-18 through 5-20 provide the annual extreme fastest-mile speed 30 ft (9.1 m) above ground for the 25-year, 50-year, and 100-year recurrence intervals, respectively. However, these extreme values are seldom of interest in computing wave characteristics in reservoirs. Generalized wind velocity-duration relations are considered to be fairly representative of maximum values that are likely to prevail over a reservoir in generally a single direction for periods up to 6 hr (excluding projects located in regions that are subject to severe hurricanes or orographic wind-flow effects). Special studies of wind characteristics associated with individual project areas should be made when determinations of unusual importance, or problems requiring consideration of wind durations exceeding 6 hr, are involved.

f. Ratio of wind velocities over water and land areas. The wind velocities described in paragraph *d* are for over land. Under comparable meteorological conditions, wind velocities over water are higher than over land surfaces because of smoother and more uniform surface conditions. Winds blowing from land tend to increase with passage over reservoir areas, and vice versa. The relationships are not constant, but vary with topographic and vegetative cover of land areas involved, reservoir configurations, and other conditions affecting air flow. However, on the basis of research and field studies (Technical Memorandum No. 132, USACE 1962), the following ratios represent averages that are usually suitable for computing wave characteristics in reservoirs that are surrounded by terrain of moderate irregularities and surface roughness:

Fetch (F_e) in Miles	Wind ratio $\frac{\text{Over Water}}{\text{Over Land}}$
0.5	1.08
1	1.13
2	1.21
3	1.26
4	1.28
5 (or over)	1.30

15-3. Computation of Wave and Wind Tide Characteristics

a. Effective wind fetch (F_e) for wave generation. The characteristics of wind-generated waves are influenced by the distance that wind moves over the water surface in the "fetch" direction. The generally narrow irregular shoreline of inland reservoirs will have lower waves than an open coast because there is less water surface for the wind to act on. The method to compensate for the reduced water surface for an enclosed body of water is computation of an effective fetch. The effective fetch (F_e) adjusts radial lines from the embankment to various points on the reservoir shore. The radials spanning 45 deg on each side of the central radial are adjusted by the cosign of their angle to the central radial to estimate an average effective fetch. The computation procedure is shown in EM 1110-2-1414, Figure 5-33 and Example Problem 7-2. Generalized relations are based on effective fetch distances derived in this manner.

b. Fetch distance for wind tide computations. Fetch distances for use in estimating wind tide (set-up) effects are usually longer than effective fetch distances used in estimating wave heights. In as much as wind tide effects in deep inland reservoirs are relatively small, extensive studies to refine estimates are seldom justified. For practical purposes, it is usually satisfactory to assume that the wind tide fetch is equal to twice the effective fetch (F_e). If wind tide heights determined in this manner are relatively large in relation to overall freeboard requirements, more detailed analyses are advisable using methods as generally discussed in Chapter 3 of EM 1110-2-1414.

c. Generalized diagrams for wave height and wave period in deep inland reservoirs. EM 1110-2-1414, Figure 5-34, presents generalized relations between significant wave height, wave period, fetch (F_e), and wind velocities corresponding to critical durations. These diagrams were developed from research and field studies based on wind speed at 10 m (33 ft). If wind speeds are for a different level, Equation 5-12 can be used to adjust to the 10 m level (EM 1110-2-1414).

d. Wave characteristics in shallow inland lakes and reservoirs. In the analyses of wave characteristics, lakes and reservoirs are considered to be shallow when depths in the wave-generating area are generally less than about one-half the theoretical deep-water wave length (L_o) corresponding to the same wave period (T). Curves presented in Figures 5-35 through 5-44 represent relations between wave characteristics, fetch distances (in feet) and constant water depths in the wave-generating area, ranging from 5 to 50 ft (1.5 to 15.2 m) (EM 1110-2-1414).

e. Wind tides (set-up) in inland waters.

(1) When wind blows over a water surface, it exerts a horizontal stress on the water, driving it in the direction of the wind. In an enclosed body of water, this wind effect results in a piling up of water at the leeward end, and a lowering of water level at the windward end. This effect is called "wind tide" or "wind set-up." Wind set-up can be reasonably estimated for lakes and reservoirs, based on the following equation:

$$S = \frac{U^2 F}{1400 D} \tag{15-1}$$

in which *S* is wind tide (set-up) in feet above the stillwater level that would prevail with zero wind action; *U* is the average wind velocity in statute miles per hour over the fetch distance (*F*) that influences wind tide; *D* is the average depth of water generally along the fetch line (EM 1110-2-1414). The fetch distance (*F*) used in the above formula is usually somewhat longer than the effective fetch (F_e) used in wave computations, as indicated in paragraph 15-3*b*. Refer to EM 1110-2-1414, Section 3-2 for a discussion of prediction models.

15-4. Wave Runup on Sloping Embankment

a. Introduction. Most dam embankments are fronted by deep water, have slopes between 1 on 2 and 1 on 4, and are armored with riprap. Rock-fill dams are considered as permeable rubble slopes and earth-fill dams with riprap armor are considered impermeable. Laboratory tests of many slopes, wave conditions, and embankment porosity provide sufficient data to make estimates of wave runup on a prototype embankment.

b. Relative runup relations. EM 1110-2-2904 Plate 25 presents generalized relations on wave runup on rubble-mound breakwaters and smooth impervious slopes. Plate 26 provides similar curves for various embankment slopes for water depths greater than three-times wave height. The curves correspond to statistical averages of a large number of small and large-scale hydraulic model test results, and have been adjusted for model scale effects to represent prototype conditions. The relations were based primarily on tests involving mechanically generated waves and may differ somewhat from relations associated with individual waves in natural wind-generated spectrums of waves. However, general field observations and comparisons with wave experiences support the conclusion that relations presented.

c. Runup of waves on sloping embankments.

(1) If waves generated in deep water (i.e., depths exceeding about one-third to one-half the wave length), reach the toe of an embankment without breaking, the vertical height of runup may be computed by multiplying the deep-water wave height (H_o) by the relative runup ration (R/H) obtained from EM 1110-2-2904, Plate 26, for the appropriate slope and wave steepness (H_o/L_o). In this case, deep-water values of H_o and L_o should be used as indices, even though wave heights and lengths are modified by passing through areas in which water depths are less than $L_o/3$ (provided the depth is not small enough to cause the wave to break before reaching the embankment). That is, the height of runup may be computed by using the deep-water steepness H_o/L_o, whether the structure under study is located in deep water or in shallow water, provided the wave does not break before reaching the toe of the structure.

(2) If waves are generated primarily by winds over open-water areas where the relative depth (d/L) is appreciably less than 0.3, the wave heights and periods should be computed by procedures applicable to shallow waters.

(3) Waves generated by wind over open-water areas of a particular depth change characteristics when they reach areas where the constant depth is substantially less, the height (H) tending to increase while the length (L) decreases. The distribution of wave energy changes as a wave enters the shallow water, the proportion of total energy which is transmitted forward with the wave toward the shore increasing, although the actual amount of this

translated energy remains constant except from minor frictional effects. If the depth continues to decrease, the steepness ratio (H/L) increases, until finally the wave becomes unstable and breaks, resulting in appreciable energy dissipation. Theoretically, the maximum wave cannot exceed 0.78 D, where D is the depth of water without wave action. After breaking, the waves will tend to reform with lower heights within a distance equal to a few wave lengths. For most engineering applications, it is satisfactory to assume that the wave height after breaking will equal approximately 0.78 D in the shallow area and that L will be the same as before the wave broke. Plate 26 would then be entered with a wave-steepness ratio equal to 0.78/L to determine the relative runup ration (R/H), and this ratio would be multiplied by 0.78 D to obtain the estimated runup height (R). This procedure should provide conservative results under circumstances in which the distance between the point where waves reach breaking depths and location of the structure under study is long enough to permit waves to reform, and short enough to preclude substantial build-up by winds prevailing over the shallower area. More accurate values could be obtained by using this breaking height (0.78 D) and period to obtain comparable deep water values of H_o and L_o.

d. Adjustments in wave runup estimates for variations in riprap.

(1) A rough riprap layer on an embankment tends to reduce the height of runup after a wave breaks. If the riprap layer thickness is small in comparison with wave magnitudes and the underlying surface is relatively impermeable, so that the void spaces in the riprap remain mostly filled with water between successive waves during severe storm events, the height of runup may closely approach heights attained on smooth embankments of comparable slope. However, if the riprap layer is sufficiently rough, thick, and free draining to quickly absorb the water that impinges on the embankment as each successive wave breaks, further wave runup will be almost completely eliminated.

(2) The design of riprap to absorb most of the energy of breaking waves is practicable if waves involved will be relatively small or moderate, but costs and other practical considerations usually preclude such design where large waves are encountered. Accordingly, the design characteristics of riprap layers are usually somewhere between the two extremes described above.

15-5. Freeboard Allowances for Wave Action

a. Purpose. In connection with the design of dams and reservoirs, the estimate of freeboard is required to establish allowances needed to provide for wave action that is likely to affect various project elements, as follows:

(1) Main embankment of the dam, and supplemental dike sections.

(2) Levees that protect areas within potential flowage limits of the reservoir.

(3) Highway and railroad embankments that intersect the reservoir limits.

(4) Structures located within the reservoir area.

(5) Shoreline areas that are subject to adverse effects of wave action.

b. Freeboard on dams. The establishment of freeboard allowances on dams includes not only the consideration of potential wave characteristics in a reservoir, but several other factors of importance, including certain policy matters.

c. Freeboard allowances for wave action on embankments and structures within reservoir flowage limits.

(1) Wave action effects must be taken into account in establishing design grades and slope protection measures for highway, railroad, levee, and other embankments that intersect or boarder a reservoir. The design of operating structure, boat docks, recreational beaches, and shoreline protection measures at critical locations involves the consideration of wave characteristics and frequencies under a range of conditions. Estimates of wave characteristics affecting the design of these facilities can have a major influence on the adequacy of design and costs of relocations required for reservoir projects, and in the development of supplemental facilities.

(2) The freeboard reference level selected as a base for estimating wave effects associated with each of the several types of facilities referred to above will be governed by considerations associated with the particular facility. Otherwise, procedures generally as described with respect to the determination of freeboard allowances for dams should be followed, and stage hydrographs and related wave runup elevations corresponding to the selected wind criteria should be prepared. However, the freeboard reference level and coincident wind velocity-duration relations selected for these studies usually correspond to conditions that would be expected with moderate frequency, instead of the rare combinations assumed in estimating the height of dam required for safety.

(3) In estimating effects of wave action on embankments and structures, the influences of water depths near the facility should be carefully considered. If the shallow depths prevail for substantial distances from the embankment or structure under study, wave effects may be greatly reduced from those prevailing in deep-water areas. On the other hand, facilities located where sudden reductions in water depths cause waves to break are likely to be subjected to greater dynamic forces than would be imposed on similar facilities located in deep water. This consideration is particularly important in estimating the effects waves may have on bridge structures that are partially submerged under certain reservoir conditions.

(4) Systematic analyses of wave effects associated with various key locations along embankments that cross or border reservoirs provide a practical basis for varying design grades and erosion protection measures to establish the most economical plan to meet pertinent operational and maintenance standards.